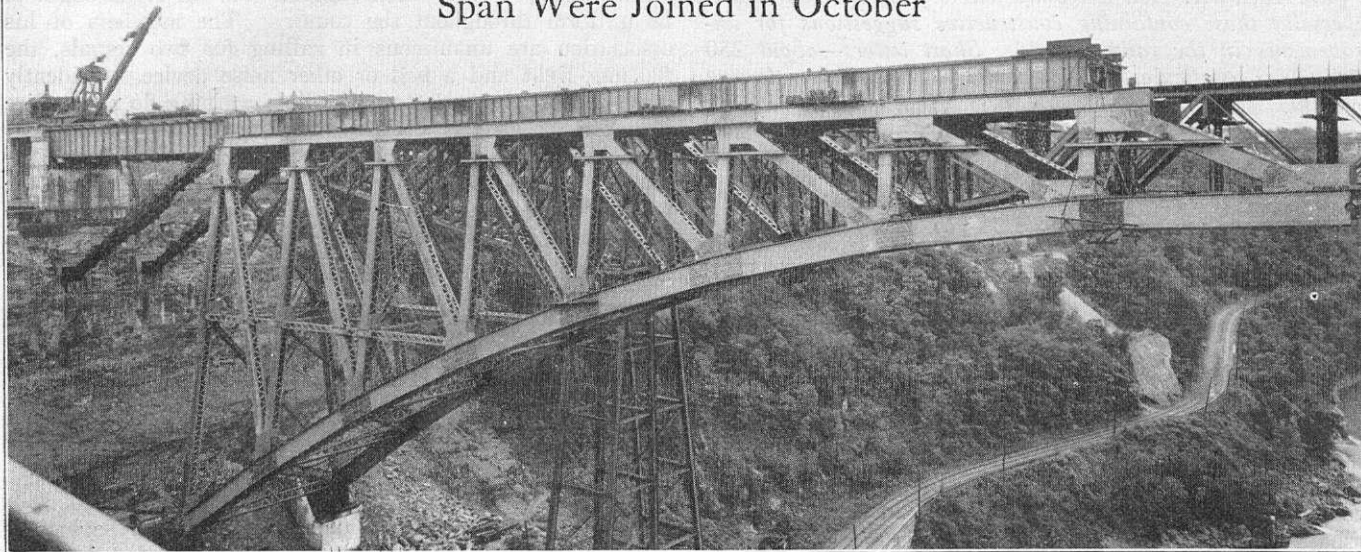


New Niagara Gorge Arch Nearing Completion

The Two Cantilever Arms of the Michigan Central's 640-Ft. Span Were Joined in October



ON OCTOBER 11, after the closing members at the crown of the Michigan Central's new 640-ft. arch across the Niagara gorge had been placed in position, the two halves of the structure, which had been extended from the two sides of the gorge by cantilever erection, were brought to bearing by releasing the backstays and the arch became a self-supporting structure. This bridge, which has the distinction of being the longest railway arch bridge in America except one, the great Hell Gate arch, has been built to replace the Michigan Central cantilever bridge erected in 1883, increased train loadings having rendered the old structure inadequate. A description of the design and details of the new bridge and an account of the preliminary engineering studies and foundation investigations appeared in the *Railway Age* of June 13, 1923, page 177. The following article is therefore confined to an account of the construction work on this bridge, which entailed the solution of many interesting problems.

General Description of the Bridge

The new bridge is located in the space between the old cantilever structure and the 550-ft. arch used by the Canadian National (Grand Trunk). It has a span of 640 ft. from center to center of hinge pins and a rise of 105 ft. It is designed to carry two railway tracks on the deck at 13 ft. centers. It belongs to the spandrel braced type of steel arch with the lower chord conforming to a parabolic curve and the top chord horizontal. It was designed for erection by the cantilever method with a pin bearing between the two halves of the lower chord at the crown so that it is a three-hinge structure for dead load exclusive of the weight of the ballast and the track construction. However, as the crown joint in the top chord will be riveted solid, the structure will function as a two-hinged arch for live load.

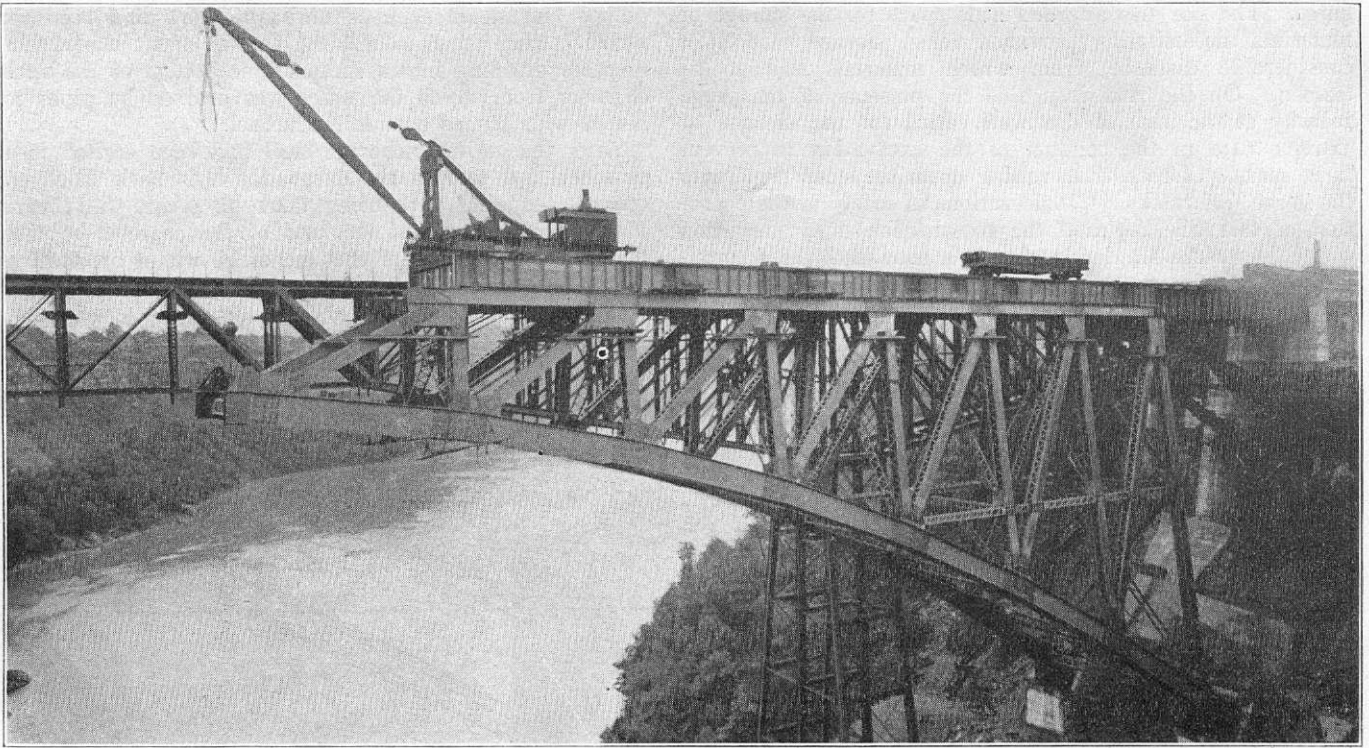
The design follows the usual practice for riveted structures except that pin connections are provided at each end of the first two vertical web members on each side of the crown. These posts are so short that deformations occurring in the arch under load or temperature changes would have introduced excessive secondary stresses if they had been provided with rigid connections to the chords.

The size and general characteristics of the main truss members are given in one of the accompanying drawings. The bottom chords are of closed box section with two webs, 3 ft. 2 $\frac{3}{4}$ in. center to center by 5 ft. 6 $\frac{1}{2}$ in. deep, each web consisting of four plates $\frac{5}{8}$ in. thick. The angles are 8 in. by 8 in. by $\frac{3}{4}$ in. throughout, the variation in the makeup being made by changing the thickness of cover plates. The maximum bottom chord section has a gross area of 714.78 sq. in., to provide for a maximum resultant stress of 9,700,000 lb. The joints in the bottom chord members are necessarily made at the panel points and the abutting ends are milled for bearing on only the middle 2 ft. 6 in. of the total depth of 5 ft. 6 $\frac{1}{2}$ in. These chord splices have been proportioned for 100 per cent riveting. The two trusses are set on a batter of 8 vertical to 1 horizontal, all members of the trusses, including the top chord, conforming to this batter.

The Floor Is an Independent Structure

For a number of reasons, among which is the possibility of a subsequent raise in the grade of the tracks across the structure, the entire floor system is essentially independent of the trusses. The design provides for a ballasted floor with the ballast-retaining construction carried on a $\frac{3}{8}$ -in. floor plate resting on 15-in. I-beams spanning longitudinally between floor beams spaced transversely at intervals of 12 ft. 2 in. These floor beams are carried by two side girders which are supported on the trusses at the panel points by means of beveled castings which compensate for the inclined position of the top chords.

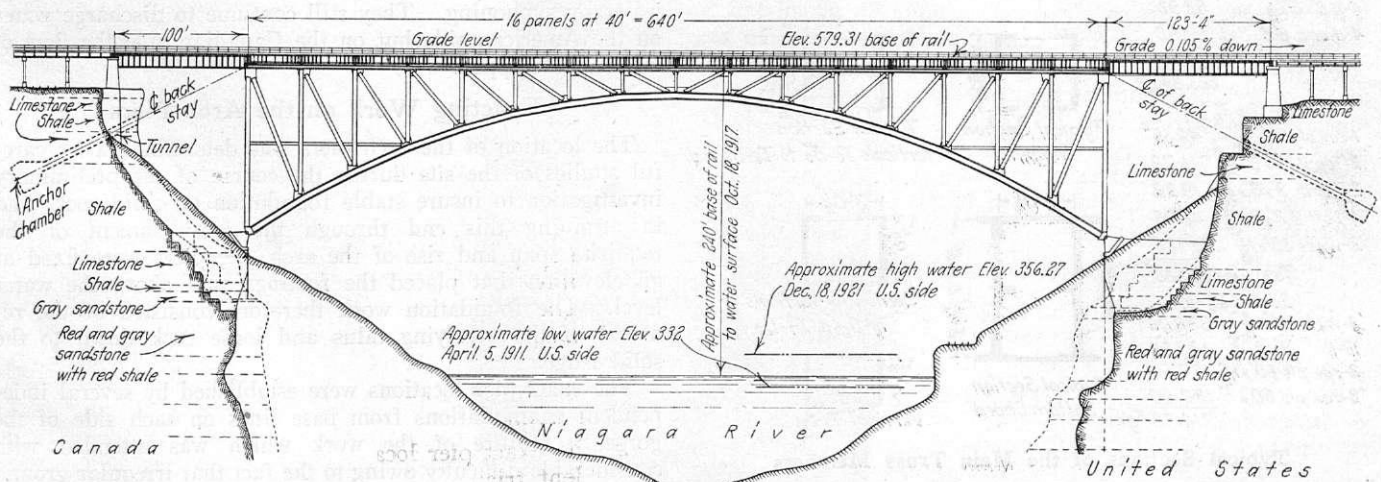
The arch bridge structure is flanked by a plate girder approach span at each end, that on the Canadian side being 100 ft. long and that on the American side 125 ft. long. These approach spans are provided with ballasted floors of similar construction. Beyond the approach spans at each end of the bridge the railway approaches consist of a number of spans of reinforced concrete viaduct terminating in street subways which carry the tracks over streets flanking the river on both sides. A total of 16,100,000 lb. of structural steel was required for the project, of which 9,600,000 lb. is in the arch trusses and bracing, 2,700,000 lb. in the arch



floor, 1,500,000 lb. in the approach spans and 700,000 lb. in the two street subways, while 1,600,000 lb. was required for the backstays, anchorages and other temporary work.

The conditions at the site of this bridge offered no alternative to the erection of the arch by the cantilever method, which was employed in the building of all of the bridges across the gorge except the original suspension bridges. The method used for this structure, however, is unique in that it was carried out without the use of anchor arms or counter

dian side. Owing to uncertainty concerning the condition of the rock to be encountered in excavating the tunnels for the backstay anchorages and in preparing the foundations for the piers, it was deemed advisable to carry on the work under an arrangement with the contractors that would permit of modifications in the plans or with the volume of work according to the dictates of judgment as determined by the actual conditions encountered. Accordingly, the work was awarded on a cost-plus-sliding-profit form of contract, the



The New Arch in Relation to the Gorge

weight structures. Instead backstays were carried into anchorages imbedded in the solid rock of the bluffs, dependence being placed on the weight and strength of the rock ledges to resist the pull on the backstays.

The Substructure Work Was Difficult

The contracts for the sub-structure were let on May 9, 1923, to the Gass-Thurston Company of Detroit, Mich., for the work on the American side and to the Federal Construction Company of Toronto, Ont., for the work on the Cana-

dian side. Owing to uncertainty concerning the condition of the rock to be encountered in excavating the tunnels for the backstay anchorages and in preparing the foundations for the piers, it was deemed advisable to carry on the work under an arrangement with the contractors that would permit of modifications in the plans or with the volume of work according to the dictates of judgment as determined by the actual conditions encountered. Accordingly, the work was awarded on a cost-plus-sliding-profit form of contract, the

Physical conditions at the site imposed a number of formidable obstacles to the prosecution of the construction, much of which involved operation on the steep faces of the

gorge. The site also afforded little space for the storage of materials, necessitating provision for a storage yard at a considerable distance, from which materials had to be teamed. On the American side the presence of the gorge railway at the base of the bluff called for the exercise of extreme care in the conduct of the excavation to prevent large masses of rock from rolling down the steep slope onto the gorge line tracks. Considerations of safety in the operation of this line required the construction of a protection over these tracks similar in plan to a snowshed, the accumulation of earth on the roof of this structure providing an effective cushion to break the fall of occasional rocks.

The preparation of the site for the tunnels and the piers for the ends of the approach spans by the removal of the overhanging ledges of rock at the tops of the bluffs disclosed a fissured and broken condition of the rock face which was much more severe than had been anticipated as a result of the preliminary investigation. On the Canadian side this condition was met as regards the approach span pier by extending one corner of the pier far down the slope. On the American side it was found necessary to set the pier 25 ft. further back on the ledge with a corresponding increase in the length of the approach span. For the same reason the tunnels for the anchorage had to be driven to a greater depth than planned to insure that the anchorage

pumps and steam syphons until the work had been completed. After much trouble the flow of water was confined to pipes draining into a sump at the bottom of the anchor chamber from which the water was drained by pipes connected with pumps outside the tunnel.

After the entire anchorage steel had been erected in the chamber and tunnels the excavation was back filled with concrete, especial care being taken to secure full bearing of the concrete against the roof of the chamber to insure that the uplift action of the anchorage would be effectively transferred to the overlying rock. This was effected by

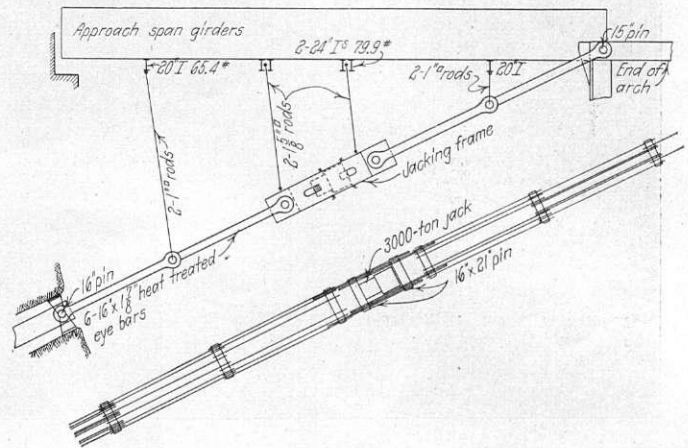
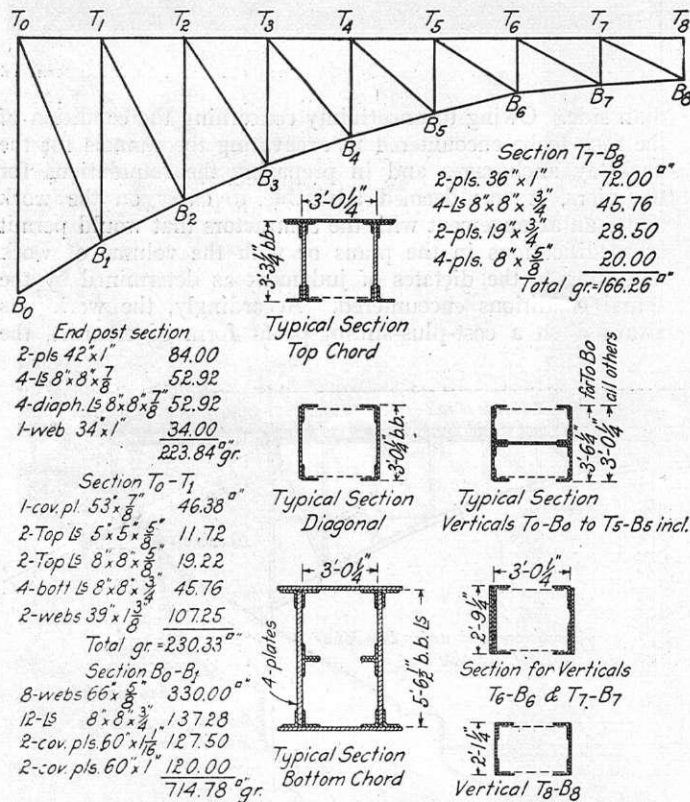


Diagram of the Back Stay Structure



Typical Sections of the Main Truss Members

would be imbedded in solid rock. The tunneling operations involved the driving of four shafts 7 ft. wide by 6 ft. 6 in. high at a downward pitch of 26 1/2 deg. from the horizontal for a distance of 82 ft. on the Canadian side and 105 ft. on the American side and terminating in anchorage chambers approximately 20 ft. deep, 20 ft. high and 17 ft. wide.

In addition to the difficulties attending this work, which were described above, the tunneling operations were subjected to a further obstacle by the opening of water bearing fissures in the rock, discharging approximately 40 gal. of water per min., which, in the absence of any opportunity for natural drainage, required the constant operation of

grouting after the chamber had been filled with concrete. On the American side the grout was pumped in through well holes drilled from the surface overhead, while on the Canadian side it was piped in through the tunnel shaft. The effectiveness of the grouting was clearly demonstrated by the appearance of the grout in the roofs of the tunnels at considerable distances from the chamber. The drain pipes from the sump at the bottoms of the chambers were retained in place to insure the continuous removal of the seepage water by syphoning. They still continue to discharge water on the American side but on the Canadian side the flow of water has stopped.

Exacting Work on the Arch Piers

The location of the arch piers was determined after careful studies of the site during the course of the preliminary investigation to insure stable foundation on solid rock, and in attaining this end through the establishment of the requisite span and rise of the arch, the piers were fixed at an elevation that placed the footings well above the water level. The foundation work therefore consisted of the removal of the overlying talus and loose rock down to the solid ledge.

The exact pier locations were established by several independent triangulations from base lines on each side of the gorge, a feature of the work which was attended with considerable difficulty owing to the fact that irregular ground made it impossible to lay the base lines horizontal. The accuracy of the locations was checked by means of a tape line supported across the river inside of a pipe line suspended from the cantilever bridge. It was again checked by the engineers of the erecting contractor with a 1,000-ft. tape swung across the stream. However, after considering the various factors tending to vitiate the accuracy of the tape measurements under the unfavorable conditions imposed it was concluded that the triangulations insured much more accurate results.

By far the most exacting feature of the pier construction was the finishing of the concrete skew backs to receive the granite coping, the setting of the coping stones and the

dressing of these stones to the required degree of accuracy with respect to location and surface. The specifications permitted a tolerance in the granite surfaces of only 1/16 in., as the correction of irregularities of contact between the skew back castings and the masonry was limited to the adjustment that could be obtained with a joint of 1/16 in. sheet lead. In carrying out this work the stones were set 3/4 in. high and then dressed down by stone cutters, the final cut requiring a "10 cut" finish to come within the tolerance limit. This work entailed the services of a force of stone cutters for a number of weeks.

The concrete work, including the concrete trestle approaches, involved the placing of 14,000 cu. yd. of concrete, of which 5,700 cu. yd. was in the arch piers and 1,500 cu. yd. in the anchor pits and tunnels. The largest single unit was the south arch pier on the United States side which contained 2,162 cu. yd. Of this total, 1,743 cu. yd., comprising the neat work above the footing, was placed in continuous run.

Noteworthy Features of the Erection

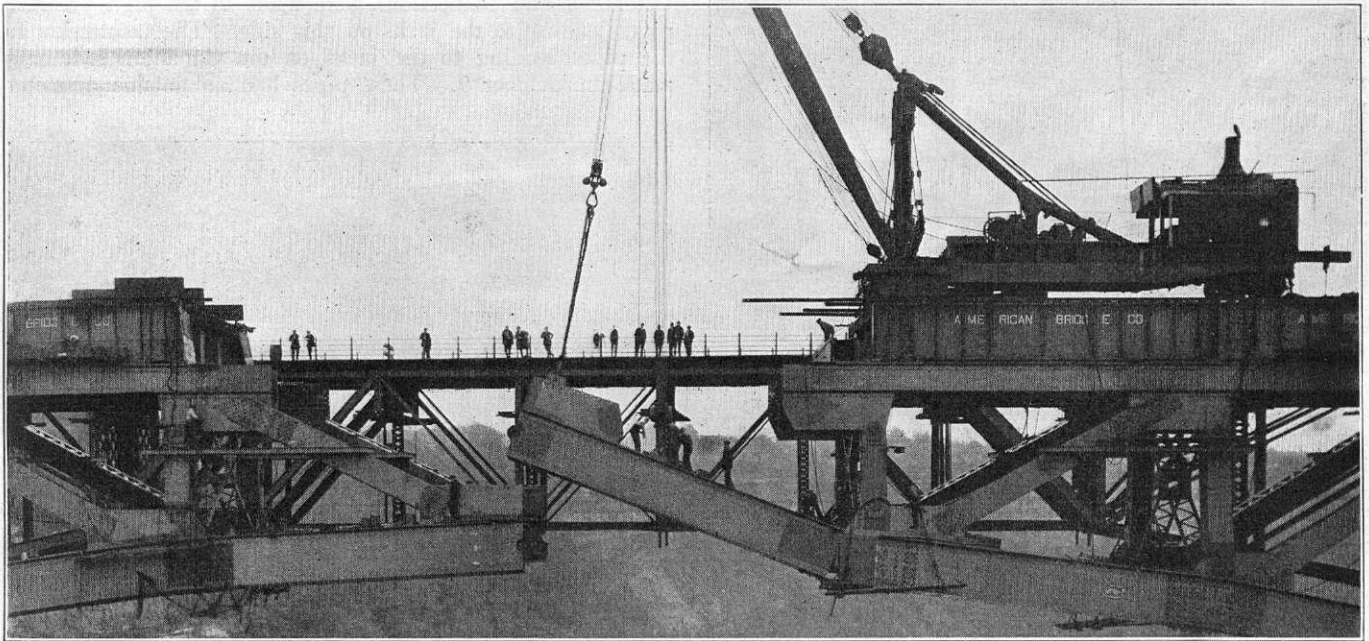
The contract for the superstructure was awarded to the American Bridge Company and covered both the fabrication and the erection. The first step in the erection of the steel was the placing of the spans for the street subways in the approaches at each end of the bridge. Following this the

prohibitive expense in view of the fact that some of the bents would have had to be over 100 ft. high.

The problem was solved by placing the approach girders first, providing a temporary outer support for them in the form of a temporary bent consisting of the posts for the second panel point (Canadian side) of the arch together with the cross bracing and a temporary base section for the columns. This bent was erected 84 ft. out from the approach pier on the American side and secured in place by struts and guys, after which the approach girders were set in place. This operation required the employment of two derrick cars and a carefully developed plan of procedure owing to the length and weight of the girders to be handled. The 125-ft. girders on the American side weighed 84 tons each.

With the girders in place the erecting equipment moved out on the span as far as the temporary bent and erected the skew back bearings, the end posts and the cross bracing, as well as the cross girder at the top, so that the approach girders could be brought to bearing upon their permanent outer support. This procedure was carried out first on the American side and after the temporary bent could be released and transferred across the river, the operation was repeated on the Canadian side.

The next step was the erection of the backstay system complete, including the suspender rods by means of which



Placing the Closing Member in Bottom Chord of the North Truss

anchor girders and the tunnel sections of the backstays were installed, after which operation had to be suspended pending the back filling of the tunnels and anchor chambers with concrete.

The procedure for commencing the erection of the two cantilever arms of the arch presented an intricate problem owing to the presence of the long approach spans at both ends of the arch. It was out of the question to place the skew-back bearings and the end posts of the arch trusses with the derrick cars standing on the concrete trestle approaches back of the approach span piers, because the reach was too great. On the other hand, to place the approach spans first introduced the problem of providing an outer support for them in the absence of the end posts of the trusses which were designed to carry them, while the alternative of providing falsework to enable the erection equipment to reach the ends of the arches would have involved

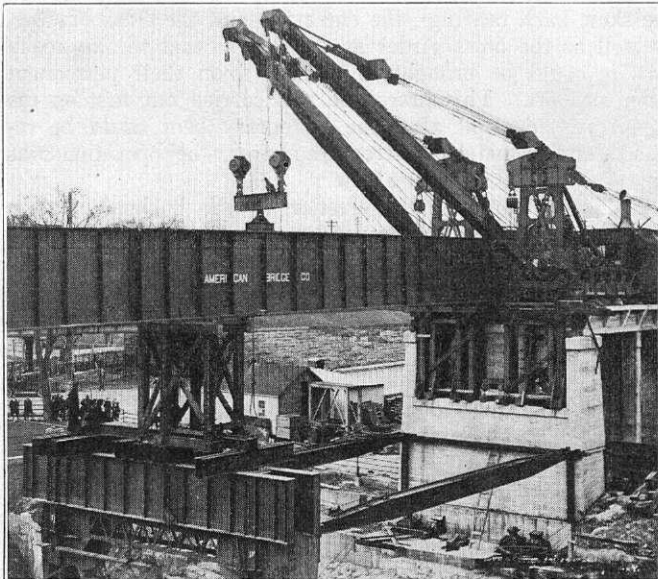
its dead weight was carried by the approach span. The weight involved was considerable, involving not only a chain of six lines of 16-in. by 1 7/8-in. heat-treated eye-bars capable of taking a stress of 4,300,000 lb. for each truss, but also a jack of 3,000 tons capacity and the necessary jacking frame required for adjusting the length of the backstay in effecting the closing of the arch at the crown. Each backstay is connected to the arch by a 15-in. pin in the end of the top chord.

The erection of the two halves of the cantilever necessarily involved an adjustment of the position of the two arms so that the outer ends were 1 ft. 10 1/2 in. above the normal position when the closure was made. The calculated gap between the ends of the cantilevers at the crown ranged from 7 9/16 in. to 15 3/4 in., depending on temperature and on the exact manner in which the stress in the backstay anchorage was transmitted to the concrete in the tunnel. In addi-

tion to the backing off of the backstays thus required to bring the arms to contact, calculations indicated that an additional adjustment in each backstay of from $2\frac{3}{8}$ in. to $2\frac{11}{16}$ in. was necessary to relieve the strain on the backstays. The maximum required adjustment of each backstay at minimum temperature was computed to be 11 in. In order to provide the necessary separation of the cantilever arms at the time of closing, the end posts of the arch trusses were erected with a backward inclination amounting to $10\frac{3}{32}$ in. at the top. This in turn required the approach span girders to be pulled back the same amount behind their normal positions.

How the Arches Were Erected

The erection of the two cantilevers proceeded as nearly simultaneously as possible with one derrick car on each



The Erection of the Approach Span Girders Was a Difficult Task

arm. As the structure was double-tracked, the cars alternated from one track to the other in placing the members in the two trusses, the members being brought forward on trucks on the other track. The normal procedure was as follows: After completing the erection of the floor for one panel, the derrick car, from its position on one of the tracks, erected the bottom chord section for the next panel on that side. Owing to the fact that the bottom chord splices are 100 per cent riveted the bottom chord members could be safely supported as a cantilever from the joint just as soon as the joint had been fully bolted and pinned and thus release the fall line for the erection of the diagonal which was placed next. Following this, the derrick car was transferred to the second track to place the bottom chord and diagonal on that side; the bottom laterals and strut and the post and top chord on that side. The derrick car was then moved to the first track for the erection of the post and top chord on that side, the cross frame and top laterals. The floor system for the panel was then erected from the same position of the derrick floor.

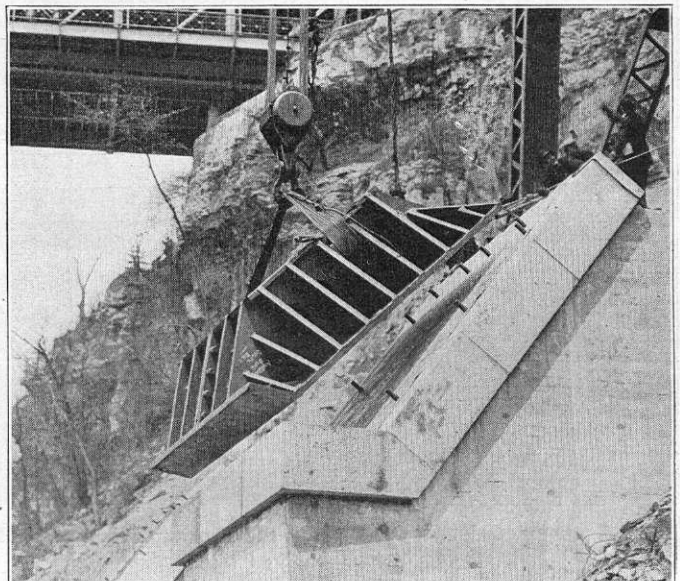
The Closing Operations

The closing gap between the ends of the completed half arches was accomplished by means of four hydraulic jacks, one for each backstay. The capacity of each of these jacks was 3,000 tons and they had been tested in the storage yard at the bridge site to 2,500 tons, which was about 25 per cent more than the required load. A jack was located at the center of each backstay between two pairs of sliding

plates, of which one pair was in fixed connection with the anchorage, by way of the lower half of the backstay, and the other pair of plates was connected to the end of the top chord by way of the upper half of the backstay. Two pins, specially designed to bear on the plunger end and the base of the jack, transferred the stress from the jack to the plates. The upper of these pins was fixed to the plates attached to the anchorage and was sliding in slotted holes in the plates attached to the end of the top chord. The sliding condition for the lower jacking pin was the reverse.

At the time the backstay was erected the slotted holes in one pair of sliding plates were filled out to pin bearing with $\frac{1}{2}$ in. shims, there being 28 shims in each plate, so that the jacking pin at this point transferred the backstay stress directly to the other sliding plate until the jack was put into operation. The jack then became a strut between the two jacking pins and first separated them sufficiently to release the bearing on the shims, which were then removed one by one while the plunger was retracted upward with the upper sliding plates, the lower pin and the shims, so that at no time was there a gap of more than $\frac{5}{8}$ in. between the fixed plate and the shims.

The pumping engine for operating the four jacks, the boiler for furnishing steam to the pump, and the control levers and water connections to the four pipe lines leading to the jacks had been rigged up on a flat car which was moved out to panel point T7 on the American side on October 6 and connections were made to the high pressure pipes leading to the jacks on this side. The connection to the pipes leading to the jacks on the Canadian side was made on October 9. These pipes had an outside diameter



Setting the Lower Bearing Casting on One of the Arch Pier Skewbacks

of one inch and an inside diameter of $\frac{1}{4}$ in. The pump, which was specially designed for this job, was a double-acting, steam driven pump with four pistons, one for each of the four pipe lines leading to the jacks. Each of these pipe lines was fitted with a mercury gage on the connection to the pump registering the pressure on each jack and the control was so arranged that the jacks could be operated simultaneously or each separately as required.

Telephonic connection was maintained between the engineer having charge of the removal of the shims. From the outer sliding plates in the backstays at each end of the bridge and the engineer in charge of the control of the pump at the center of the bridge. The engineer at the

center of the bridge also had a full view of the ends of each half span, so that he was in direct control of all operations. Two valves were provided at each jack, one designed to act instantaneously and the other a safety valve with a $1/32$ in. opening designed to release the pressure gradually. This was later found too slow and after having used it in testing out the piping and connections, it was removed for the final closing operation. The operating pressure was a little above 3,000 lb. per sq. in. The diameter of the jack plunger was 39 in., giving a total jack pressure of 3,600,000 lb., equal to the pull in the backstay after the derrick car had been moved off the bridge.

At 3:20 p. m., October 9, the American side was tried out by taking out one shim but on account of the small opening of the safety valve the half arch had only moved a fraction of an inch by 4:30 o'clock. The safety valves were then removed and the lowering of the American half of the arch was continued. By 5:30 p. m. the end of the half span had been lowered $4\frac{1}{2}$ in. and moved westward two inches. Lowering of the Canadian half of the arch was started on October 10 at 7 a. m. and by 9 o'clock, four shims had been removed. The jacks on the American side were then put into operation and both halves of the arch were lowered at the same time until 11:30 a. m. There was then one inch clearance between the center pin and the pin hole at the crown of the arch. The operation of the jacks was then stopped and the ends of the trusses were



An Early Stage in the Erection of the Arch on the United States Side

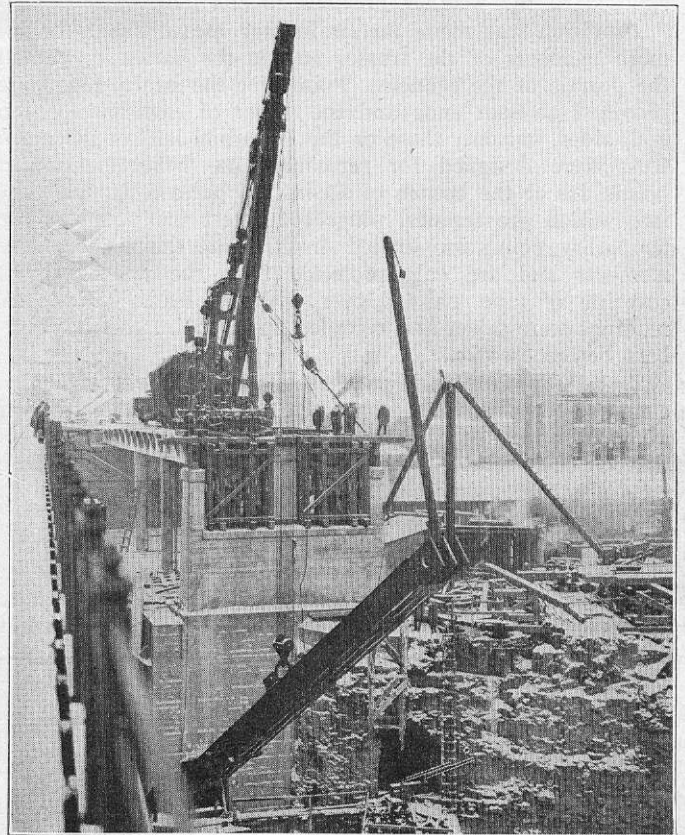
lined up with little effort. As soon as this was done the placing of the splice plates at the center joint of the bottom chord was started. These plates were very difficult to enter and the work had not been finished by 6 p. m.

The placing of the splice plates at the center joint and bolting them to the Canadian half of the arch was resumed on the following morning.

The bottom chord bracing was also placed in panel B7 B8 of the U. S. half of the arch, which was connected only at panel point B7. Lowering was resumed in the afternoon but in pumping up the jacks a leak was found on the Canadian side and it was 4 o'clock before the final closing operation was started. The two halves then moved together at a rate of $\frac{1}{2}$ in. in 6 min. When the pins

were bearing and the pressure had dropped to 2,100 lb. per sq. in. the lowering operation was again stopped, holding the pressure on the jacks, while the lateral bracing was connected and the splice plates at the center joint were being bolted up, using 1-in. bolts in the $1\frac{1}{4}$ -in. holes in the American side of the joint. By 5 o'clock this was finished and the slacking off of the jacks continued. By 5:05 the pressure on the jacks had dropped to 1,800 lb. per sq. in. and by 5:10 the pressure was zero and the structure had become self-supporting as a three-hinged arch.

While the closing operation took considerable time the whole operation was very satisfactory and the movements and deflections followed the calculated movements very closely. By October 15 the center posts and the two center



Installing a Part of the Back Stay in the South Tunnel on the Canadian Side. The North Tunnel Is Seen at the Right

panels of the top chord of the arch truss and the lateral sway bracing had been placed and everything was ready for changing the arch from the three-hinged to a two-hinged condition. For this purpose a screw 7 in. in diameter by 6 ft. long, provided with two nuts at each end, bearing on diaphragms riveted to the top chords on each side of the center joint had been furnished. The closing temperature desired was 60 deg., which was obtained at 4 p. m. on October 15, making it necessary only to screw the nuts up tight against the diaphragms and proceed with the drilling of the rivet holes which had been left blank in the top cord and in the gusset plate connecting the center post at B8 for this purpose on one side of the center joint. These holes have since been drilled, the rivets have been partly driven and the floor has been erected on the center panels so that the steel work is practically complete except for riveting.

The trusses were laid out at the shop with a camber equal to the deflection under dead load plus one-half of the live load on the whole length of the bridge. The full riveted splices in the bottom chord necessarily entailed long, heavy

splice plates on both sides of the webs, but this did not lead to any difficulty in making the joints in erection.

The necessity for long rivet grips in many of the splices was made the subject of considerable study with particular reference to the difficulty of securing tight rivets and the tendency of the joint to "pack out" due to the thickness of the paint coating between the plates. A series of tests made by the railroad showed that the value of the interplate painting is largely destroyed by the heat of large rivets which burn out the oil, leaving only the inert pigment. This led to the decision to substitute a special coating known as "Hipo" oil for the shop coat on all field splices. This material gives a much thinner film over the plates, thereby favoring a much tighter packing of the splices, yet affording adequate protection to the steel for the short time that it is exposed to the weather previous to erection.

Provision was made for strain-gage measurements on all main members of the trusses and in the backstays and at the portals of the tunnels. Points for the strain gage were provided at both ends and the center of each member on both sides, but only those on the Canadian half of the north truss were designed for permanent use. The permanent points are at the bottom of $\frac{3}{8}$ -in. tap holes $\frac{5}{16}$ in. deep, into which are screwed plugs to protect the points. The temporary points are drilled directly into the metal of the members and are only protected from the weather by a covering of tape, painted over with red lead. "No-load" readings were taken and recorded on all points of the members before erection.

In addition to strain-gage readings on the backstays, a complete set of measurements was taken of the position of all panel points in three dimensions after the complete erection of each added panel in the cantilever arms. These were checked against charts showing the calculated position of each point for the corresponding temperature and loading. To eliminate from these measurements the warping effect of unequal temperature on the two sides of the structure on account of sunlight, all measurements and readings were taken as early in the morning as conditions of light would permit.

The design of this bridge was developed under the direc-

tion of H. Ibsen, bridge engineer of the Michigan Central, assisted by C. L. Christensen, assistant bridge engineer. The project has been handled under the general direction of J. F. Deimling, chief engineer of the Michigan Central at Detroit, Mich. Olaf Hoff, consulting engineer, 50 Church street, New York City, has acted as advisory engineer on the project. The construction has been carried out under the direction of J. H. Curtin, resident engineer, and under the general supervision of Mr. Ibsen.

Annual Accident Bulletin for 1923

THE INTERSTATE COMMERCE COMMISSION has issued Accident Bulletin No. 92, consisting of over 100 large pages, containing the record of collisions, derailments and other accidents occurring on the railroads of the United States during the 12 months ending with December 31, 1923.* The total number of casualties in the tables for 1923 is 179,097; made up of 7,385 persons killed and 171,712 injured. The various classes of accidents and casualties are analyzed in great detail, as in preceding years, with minute classification of causes of accidents. Most of the totals of 1923 are larger than those of 1922. The business done by the railroads was greater, as measured by the number of locomotive miles, which in 1923 aggregated 1,813 millions; while in 1922 the total was 1,600 millions; and, as stated in the bulletin, the "increased exposure" in 1923 is generally reflected in the statistics.

To this, however, there is an exception, the item of passengers killed and injured. The decrease in passenger fatalities is called "a conspicuous feature," the total, 138, being the lowest on record, though the number of passenger miles was 7.2 per cent more than in 1922. A table is given showing the total number of passengers killed from all causes, for a series of years. A part of this table is quoted below (Table D) the figures taken from the bulletin being shown in column A. The number of passengers killed in

* The last annual statistical bulletin, No. 87, was noticed in the *Railway Age* of September 29, 1923, page 594, and the one preceding that on October 7, 1922, page 652.